Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1237

ASSESSMENT OF THE NATIONAL BUILDING CODE OF CANADA SEISMIC DESIGN REQUIREMENTS FOR OPERATIONAL AND FUNCTIONAL COMPONENTS USING FLOOR RESPONSE SPECTRA

Murat Saatcioglu[1], Mohammad Shooshtari[2], Simon Foo[3] and Nove Naumoski[4]

ABSTRACT

Previous earthquakes have demonstrated that the majority of injuries, fatalities and property damage during strong earthquakes are caused by damage to operational and functional components (OFC) of buildings. These components include architectural elements, mechanical and electrical equipment, and building contents. The current National Building Code of Canada (NBCC 2005) addresses seismic design of non-structural components through an empirical approach. An expression is provided for the computation of horizontal force for which the component is designed. A more rational approach for designing these elements against seismic excitations involves the use of floor design spectra. An analytical investigation was carried out by the authors to compile data for floor response spectra. This paper presents the comparisons of the NBCC (2005) requirements for OFCs used in Canada with those based on floor response spectra.

Introduction

Performance of building response during previous earthquakes has clearly indicated the vulnerability of operational and functional components (OFC) to seismic damage and life safety. OFCs include architectural components, such as parapets, claddings, partitions, stairways, lighting systems, suspended ceilings; mechanical and electrical equipment, such as pipes and ducts, escalators, central control panels, transformers, emergency power systems, fire protection systems, machinery; and building contents, including furniture, storage racks, book shelves, cabinets, etc. Earthquake reconnaissance reports consistently indicate poor performance of OFCs during seismic excitations, claiming more injuries, fatalities, property and financial losses than those inflicted by structural damage (McKevitt et al. 1995). As an example, it was reported by McKevitt (1995) that approximately 20% of deaths during the 1994 Northridge Earthquake occurred because of poor performance and failure of OFCs. In Canada, the 1988 Saguenay earthquake, the strongest event in eastern North America recorded within the last 50 years, caused very little structural damage, but resulted in injuries, property damage, and economic loss

---

[1] Professor and University Research Chair, Department of Civil Engineering, University of Ottawa, Ottawa, ON
[2] Assistant Professor, Department of Civil Engineering, Bu-Ali Sina University, Hamedan, Iran
[3] Risk Management Specialist, Public Works & Government Services Canada, Gatineau, PQ
[4] Adjunct Professor, Department of Civil Engineering, the University of Ottawa, Ottawa, ON

associated with the failures of OFCs in buildings. There have been many incidences that a building, which sustained only minor structural damage, was deemed unsafe and unusable as a result of extensive damage to its OFCs.

Equipment failures and the debris caused by falling objects could critically affect the performance of vital facilities such as emergency command centers, fire and police stations, hospitals, power stations and water supply plants. Failure of such components also poses serious problems for search and rescue operations after an earthquake, resulting in additional and unnecessary increases in casualties.

Previous experience with OFC performance prompted research in the past that has led to the development of design and analysis techniques for such components. The International Building Code (ICC-IBC 2006) in the United States requires the computation of seismic forces applied on OFCs as well as maximum lateral deflections to ensure survivability of these components. In Canada, the Canadian Standards Association (CSA) published the first edition of CSA S832 ”Guideline for Seismic Risk Reduction of Operational and Functional Components (OFCs) of Buildings” in 2001 (CSA S832-2001). The Standard was subsequently revised in 2004 and was published under the title of “Seismic risk reduction of operational and functional components (OFCs) of buildings” (S832-2004). The CSA 832-2004 includes two methods for the evaluation of OFC behaviour; i) prescriptive method and ii) analytical method. The prescriptive method provides general concepts for design and performance, including the details for fastening OFCs to prevent or minimize seismic movements, but otherwise relies on guidelines published by the industry for specific equipment or component manufactured. In the analytical method, forces and/or displacements of OFCs are calculated using a simple method, such as the equivalent static force method, or a refined method involving response spectrum or time history analysis. The refined methods are mandatory for OFCs with a mass greater than 20% of that of the floor or 10% of the total building mass. The National Building Code of Canada (NBCC 2005) addresses the design of non-structural components against seismic effects through an empirical approach. The approach involves the calculation of a lateral force for which the OFC should be designed. This is done by finding the product of OFC mass and design spectral acceleration at 0.2 sec period, adjusted for soil conditions, height and ductility of the element. Further details of the procedure are provided under “NBCC 2005 Requirements.”

Floor Response Spectra

Floor response spectra, compatible with the uniform hazard spectra (UHS) of NBCC-2005 were generated by the authors for reinforced concrete buildings in Canada through dynamic inelastic response history analyses of selected buildings (Shooshtari et al. 2010). A total of 12 reinforced concrete buildings, designed on the basis of the seismic provisions of NBCC-2005, were selected. The buildings consisted of 5, 10 and 15 storey heights. Both moment resisting frame and shear walls structural systems were considered as lateral force resisting systems. Consequently, three moment resisting frame buildings and three shear wall buildings with three different building heights were considered in Vancouver and Ottawa, separately. The Vancouver buildings represented structures in western Canada, and the Ottawa buildings represented structures in eastern Canada. All buildings were assumed to be located on firm soil (Class “C” in NBCC-2005). Ground motion records for dynamic analysis were selected to match the UHS given in NBCC-2005 for Vancouver and Ottawa. A total of 15 synthetic records, generated by Atkinson and Beresnev (1998), were selected for each city with a probability of occurrence of 2% in 50 years, reflecting different earthquake distance and magnitude relationships. Computer software DRAIN-2DX (Prakash, V. et al. 1993) was used to carry out nonlinear time history analysis of buildings. Damping was specified as 5% of critical damping, consisting of stiffness and mass dependent components. A total of 180 analyses were conducted, using 30 code-compatible earthquake records for Eastern and Western Canada. The analyses provided acceleration time histories for each floor of each building, resulting in 1800 floor response spectra. The spectra were then used to conduct a regression analysis to obtain floor response spectra at mean and mean plus one standard deviation. Design spectra are commonly specified at mean plus one standard deviation level (Rosenblueth 1980) in order to ensure that there is a relatively small probability that the response will be above the specified design level. The mean plus one standard deviation level corresponds to 84% of all spectral values being below the specified level. This level was deemed appropriate for the development of design spectra. Figs. 1 and 2 show spectral values for all buildings.

The examination of response spectra indicates that there is a progressive increase in response going from the first floor to upper floors. The rate of response amplification is higher in low-rise buildings as compared to companion medium and high-rise buildings. The 5-storey frame buildings showed an amplification of approximately a factor of 4 for the roof response relative to the response of the first-storey. However this amplification factor was approximately 3 and 2 for 10-storey and 15-storey buildings. It was further observed that the shear wall buildings analyzed developed higher floor amplifications than the companion frame structures having the same height. The amplification in floor response was higher for short period structures. The response spectra further indicated higher amplifications for buildings in Vancouver, which were subjected to stronger ground motions relative to those in Ottawa. A smoothened design response spectrum was then established for the roof level of each building such that it would capture most of the response amplification associated with building height. The floor design response spectra at the roof level are indicated in Figs. 1 and 2 (as dashed lines) for frame and shear wall buildings of different height, located in Ottawa (representing eastern Canadian seismicity) and Vancouver (representing western Canadian seismicity). The same figures also include the UHS specified in NBCC-2005, which may be used for the first floor response. Interpolation of spectral values between the UHS and the roof design spectrum is recommended to establish floor design spectral values for in-between floor levels. The UHS given in NBCC (2005) assumes a constant value for periods of less than 0.2 sec, equal to the UHS at T = 0.2 sec. This is a conservative assumption, which may not have much effect on building design, since there would be very few buildings with peirods of less than T = 0.2 sec. However, this is not the case for the OFCs. OFCs may be quite rigid, with periods of less than 0.2 sec. Therefore, the design floor spectral values can be lower than those associated with UHS, in the short period range, and need not exceed the value specified for the roof level. This is especially true for frame buildings.

NBCC 2005 Requirements

The current National Building Code of Canada (NBCC 2005) addresses the design of non-structural components against seismic effects by classifying them into 21 categories and by suggesting an empirical approach. These categories are given in Table 1. Accordingly, these components must be designed to accommodate building deflections, as well as element and component deflections, while resisting a lateral force V_p defined in Eq. (1), applied at the centre of mass.

$$V_p = 0.3\ F_a\ S_a(0.2)\ I_E\ S_p\ W_p \qquad (1)$$

Where, W_p is the weight of non-structural component, I_E is importance factor, F_a is acceleration-based site coefficient and S_a (0.2) is the UHS value at 0.2 sec. The horizontal force factor S_p is computed by Eq. (2), with minimum and maximum values of 0.7 and 4.0, respectively.

$$S_p = C_p\ A_r\ A_x / R_p \qquad (2)$$

where, C_p, reflects risk associated with the failure of component. This value is higher for components that contain toxic or explosive materials, and becomes 1.0 for ordinary components. A_r represents the dynamic amplification of the component associated with the proximity of the natural period of the component to that of the building. The highest amplification of 2.5 is attained when the natural period of the component is similar to the period of the building. A_x reflects the effect of building height, and is defined as $(1+2h_x/h_n)$, where h_x is the floor height under consideration and h_n is the total building height. R_p reflects the available ductility in the component, similar to the ductility related force modification factor R_d used for building design. The factors C_p, A_r, and R_p are defined in Table 1. For most OFCs, V_p is applied in the horizontal direction, except for horizontally cantilevered floors, balconies and other similar elements where the lateral force is to be applied in the vertical direction as an upward and downward force. The code also emphasizes that the lateral deflection of an OFC, computed using an elastic analysis, should be multiplied by R_p/I_E to obtain a realistic deflection within the inelastic range of deformations. For regions of low seismicity, where IEFaSa(0.2) is less than 0.35, the requirements specified above do not apply to Categories 6 through 21 specified in Table 1. Other prescriptive modifiers are assigned to certain conditions pertaining to OFCs in NBCC (2005) to modify the design force level specified in Eq. (1).

Comparisons of NBCC 2005 Requirements with Floor Response Spectra

Design force levels computed by Eq. (1) as per NBCC (2005) requirements are compared with those obtained from the floor response spectra presented in the paper, in terms of spectral values. The comparison is based on elastic force levels with $R_p = 1.0$. Two values of dynamic amplification factor were considered; $A_r = 1.0$ for components that have significantly different natural periods than those of the buildings housing them, and $A_r = 2.5$ when the natural periods are similar, resulting in signifant dynamic amplifications. Eq. (1) is then expressed in terms of V_p/W_p, with $C_p = 1.0$ for ordinary components, withouth any serious risk associated with toxic or explosive materials. The comparisons are shown in Figs. 3 and 4 for the first storey and roof levels of 5, 10, and 15 storey frame and shear wall buildings. They indicate that the NBCC (2005) design approach can not capture frequency dependent variations in response. The code expression results in lower forces than those indicated by floor response spectra of frame buildings when A_r=1.0, in the mid-period range, between 0.02 sec and 0.2 sec., especially at the first-storey level. The discrepency is wider for frame buildings in Ottawa. When A_r=2.5, the code expression over-estimates design forces in upper floors of frame buildings, relative to those obtained from the floor response spectra. The corrleation improves for shear wall buildings, though significant discrepancies exist in short and high period ranges.

Conclusions

OFCs in buildings subjected to seismic excitations can best be designed using floor response spectra associated with the seismicity of the region. The 12 frame and frame-shear wall interactive buildings designed for western and eastern Canada indicate that the NBCC 2005 requirements can not capture period dependancy of OFCs in design. This becomes more apparanet in frame buildings designed for Ottawa. While the correlation improves for shear wall buildings, the descrepency remains high in short and long period ranges, especially for upper stories. It is recommended that OFCs should be designed using appropriate floor design spectra.

References

Atkinson, G.M. and Beresnev, I.A.. (1998). Compatible ground-motion time histories for new national seismic hazard maps. Canadian Journal of Civil Engineering, 25: 305-318.

CSA-S832. (2001). Guideline for Seismic Risk Reduction of Operational and Functional Components (OFCs) of Buildings. Canadian Standards Assoc., Rexdale, ON, Canada.

CSA-S832. (2006). Standard for Seismic Risk Reduction of Operational and Functional Components (OFCs) of Buildings". Canadian Standards Assoc. Rexdale, ON, Canada, pp.104.

ICC-IBC (2006). International Building Code (IBC-2006), Int. Code Council. U.S.A., p.679.

McKevitt, W.E., Timler, P.A.M., and Lo, K.K. (1995) Nonstructural damage from the Northridge Earthquake, Canadian J. of Civil Engineering, Vol. 22, No. 2, pp. 428-437.

NBCC. (2005). National Building Code of Canada, National Research Council, Ottawa, Canada.

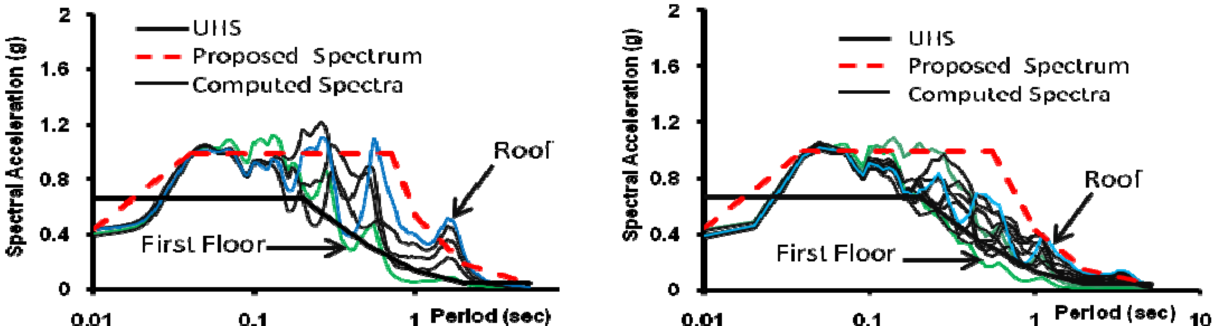
Prakash, V., Powell, G.H., and Campbell, S. (1993). DRAIN-2DX: Base Program Description and User Guide. Version 1.10. University of California at Berkeley.

Rosenblueth, E. 1980. Design of earthquake resistant structures. Pentech Press Limited, Plymouth, United Kingdom.

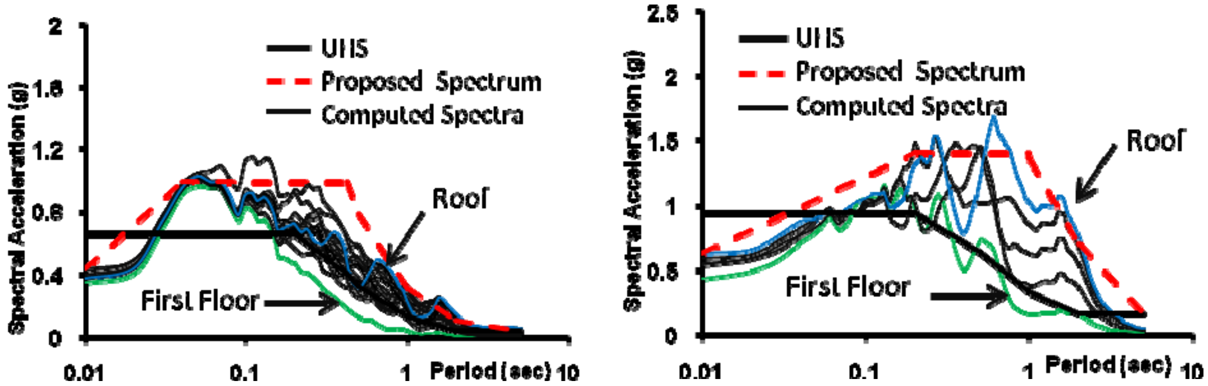
Shooshtari, M., Saatcioglu, M., Naumoski, N. and Foo, S. (2010). Floor response spectra for seismic design of operational and functional components of concrete buildings in Canada. Submitted for publication in the Canadian Journal of Civil Engineering.

Table 1. Elements of Structures and Nonstructural Components and Equipment (NBCC 2005)

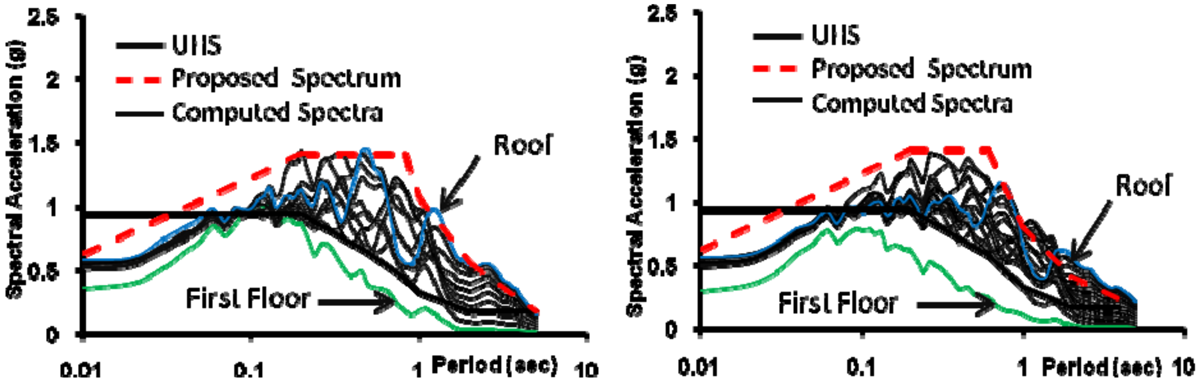
Category	**Part or Portion of *Building***	C_p	A_r	R_p
1	All exterior and interior walls except those of Category 2 and 3	1.00	1.00	2.50
2	Cantilever parapet and other cantilever walls except retaining walls	1.00	2.50	2.50
3	Exterior and interior ornamentations and appendages	1.00	2.50	2.50
4	Floors and roofs acting as diaphragms	-	-	-
5	Towers, *chimneys*, smokestacks and penthouses when connected to or forming part of a *building*	1.00	2.50	2.50
6	Horizontally cantilevered floors, balconies, beams, etc.	1.00	1.00	2.50
7	Suspended ceilings, light fixtures and other attachments to ceilings with independent vertical support	1.00	1.00	2.50
8	Masonry veneer connections	1.00	1.00	1.50
9	Access floors	1.00	1.00	2.50
10	Masonry or concrete fences over 1.8 m tall	1.00	1.00	2.50
11	Machinery, fixtures, equipment, ducts and tanks (including contents)			
	that are rigid and rigidly connected	1.00	1.00	1.00
	that are flexible or flexibly connected	2.50	1.25	2.50
12	Machinery, fixtures, equipment, ducts and tanks (including contents) containing toxic or explosive materials, materials having a flashpoint below 38ºC or fighting fluids;			
	that are rigid and rigidly connected	1.50	1.50	1.00
	that are flexible or flexibly connected	2.50	1.25	2.50
13	Flat bottom tanks (including contents) attached directly to a floor at or below *grade* within a *building*	0.70	1.00	2.50
14	Flat bottom tanks (including contents) attached directly to a floor at or below *grade* within a *building* containing toxic or explosive materials, materials having a flashpoint below 38ºC or firefighting fluids	1.00	1.00	2.50
15	Pipes, ducts, cable trays (including content)	1.00	1.00	3.00
16	Pipes, ducts (including contents) containing toxic or explosive materials	1.50	1.00	3.00
17	Electrical cable trays, bus ducts, conduit	1.00	2.50	5.00
18	Rigid components with ductile material and connections	1.00	1.00	2.50
19	Rigid components with nonductile material or connections	1.00	1.00	1.00
20	Flexible components with ductile material and connections	1.00	2.50	2.50
21	Flexible components with nonductile material or connections	1.00	2.50	1.00



a) 5-Storey Frame Building in Ottawa b) 10-Storey Frame Building in Ottawa

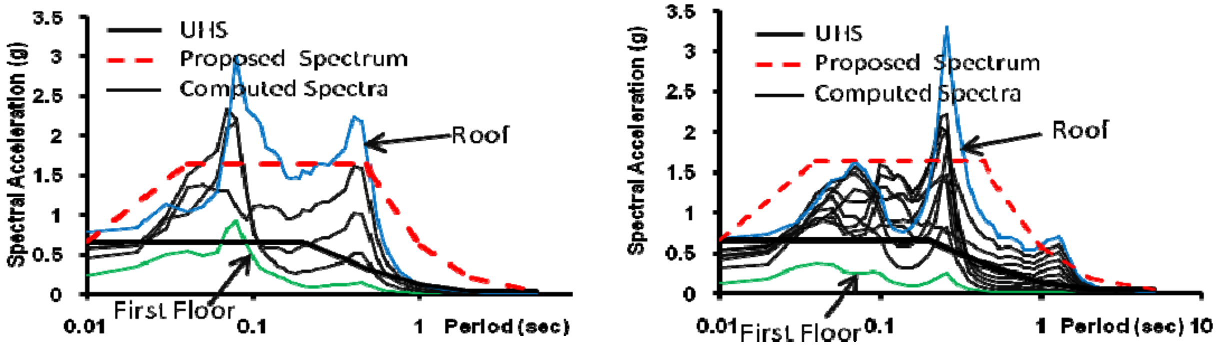


c) 15-Storey Frame Building in Ottawa d) 5-Storey Frame Building inVancouver

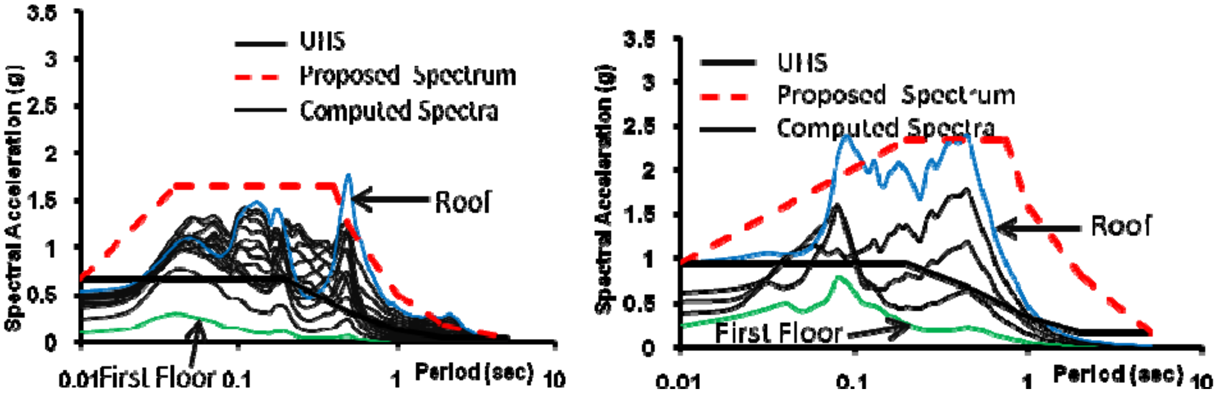


e)10-Storey Frame Building in Vancouver f) 15-Storey Frame Building inVancouver

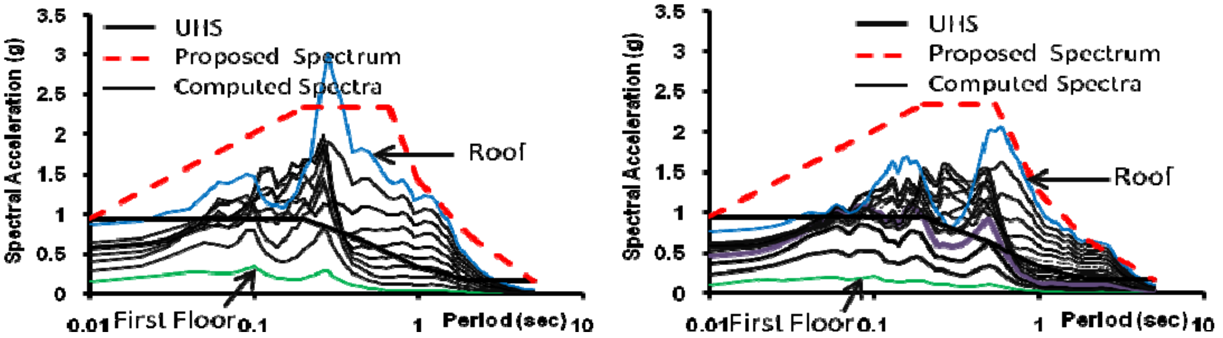
Figure 1. Floor response spectra and comparisons with UHS for frame buildings



a) 5-Storey Shear-Wall Building in Ottawa b) 10-Storey Shear-Wall Building in Ottawa

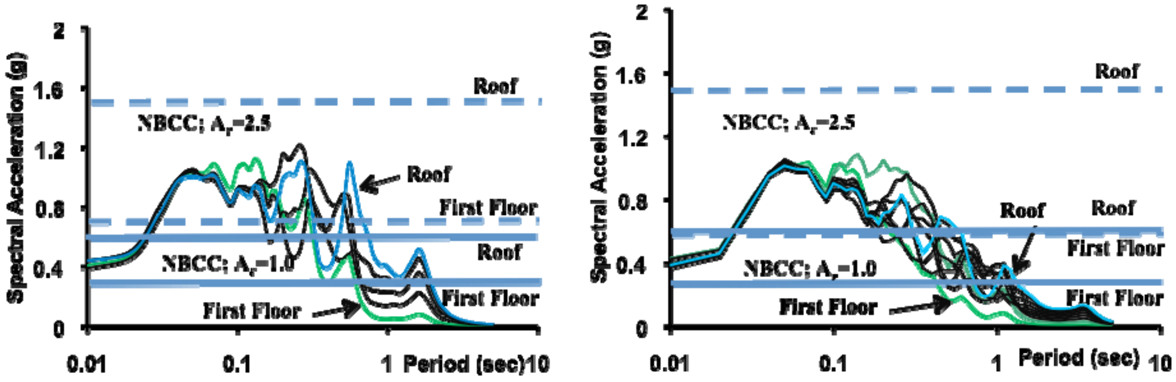


c) 15-Storey Shearwall Building in Ottawa d) 5-Storey Shearwall Building in Vancouver

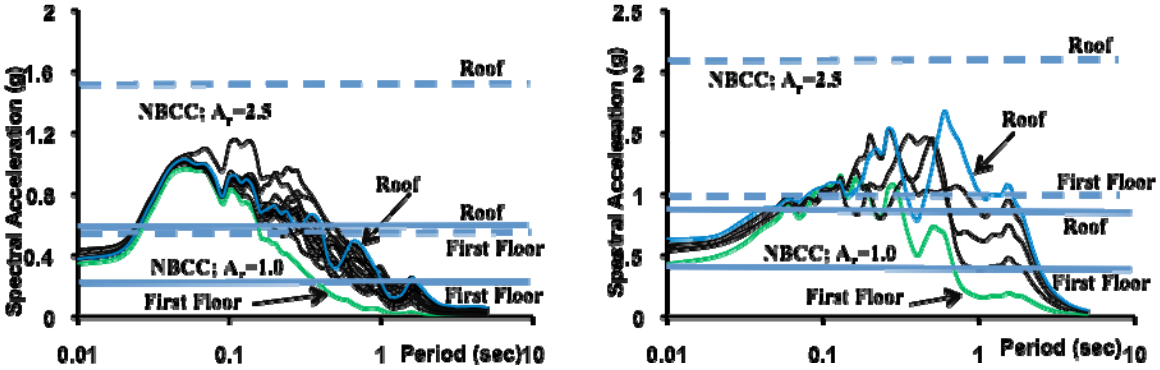


e) 10-Storey Shearwall Building in Vancouver f) 15-Storey Shearwall Building in Vancouver

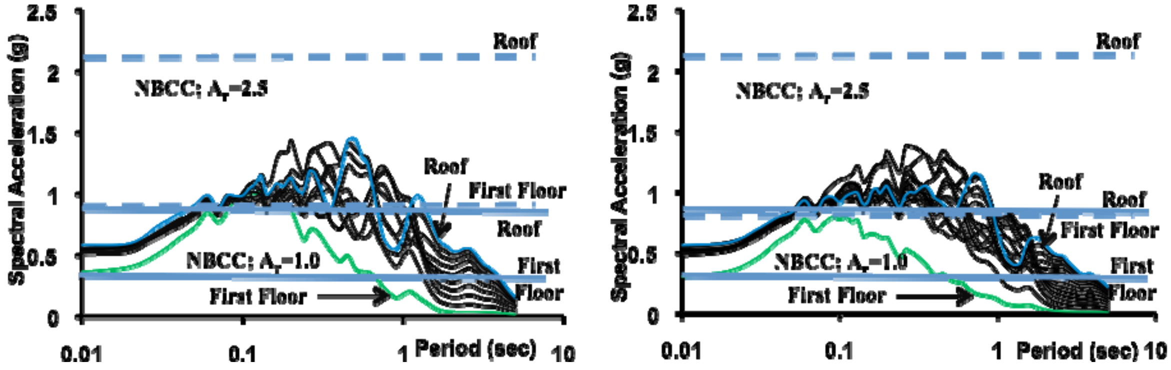
Figure 2. Floor response spectra and comparisons with UHS for shear-wall buildings



a) 5-Storey Frame Building in Ottawa b) 10-Storey Frame Building in Ottawa

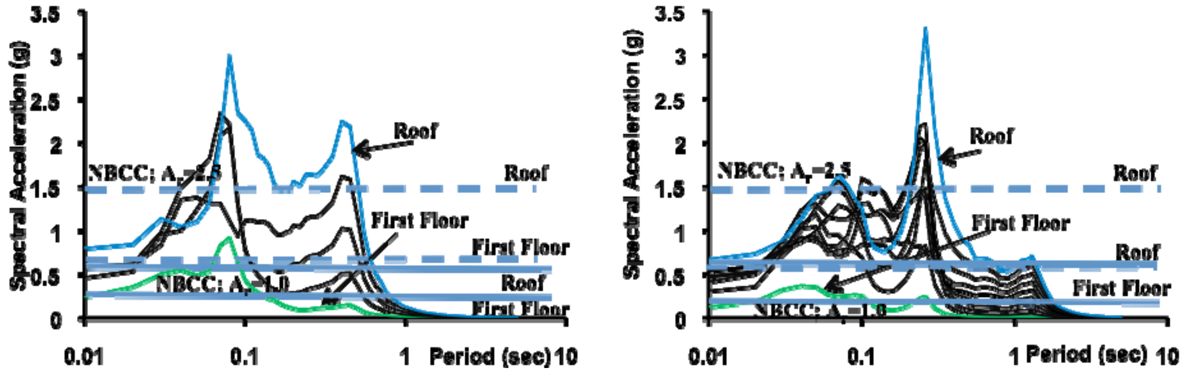


c) 15-Storey Frame Building in Ottawa d) 5-Storey Frame Building inVancouver

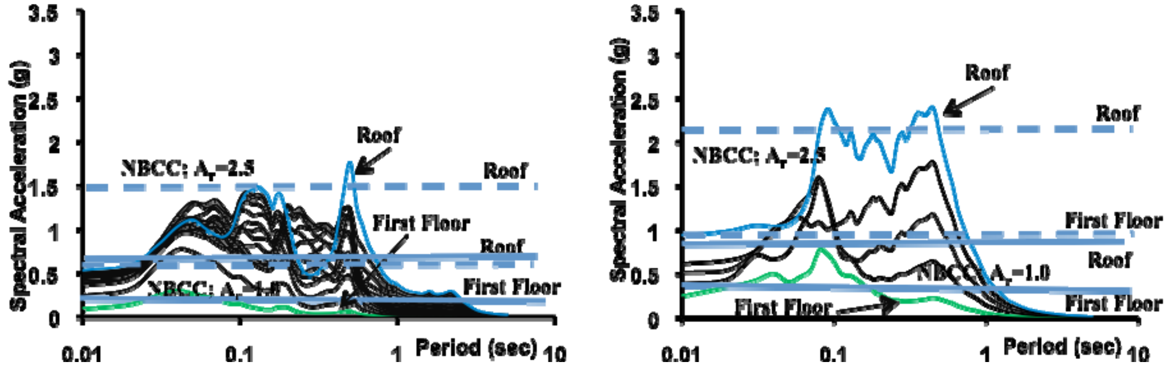


e) 10-Storey Frame Building in Vancouver f) 15-Storey Frame Building inVancouver

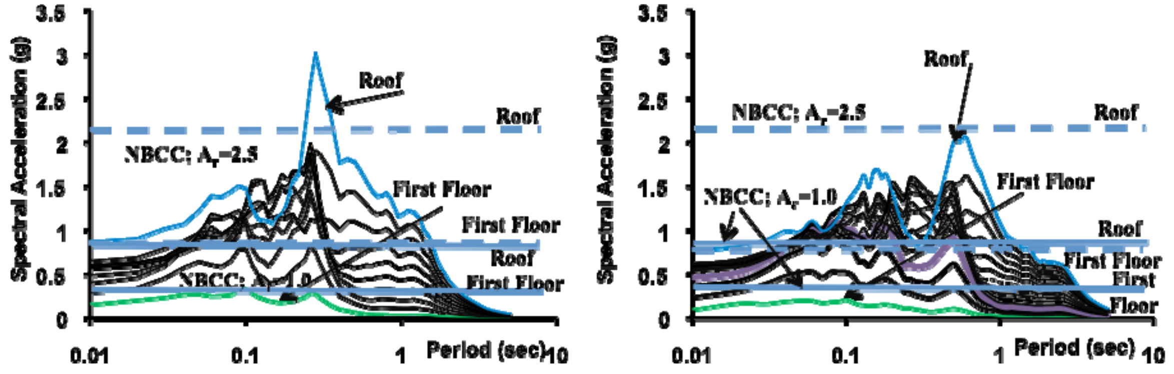
Figure 3. Comparisons of floor response spectra for frame buildings with the requirements of NBCC-2005 for OFC design



a) 5-Storey Shear-Wall Building in Ottawa b) 10-Storey Shear-Wall Building in Ottawa



c) 15-Storey Shearwall Building in Ottawa d) 5-Storey Shearwall Building in Vancouver



e) 10-Storey Shearwall Building in Vancouver f) 15-Storey Shearwall Building inVancouver

Figure 4. Comparisons of floor response spectra for shear-wall buildings with the requirements of NBCC-2005 for OFC design